NON PRISMATIC FOLDED ROOF STRUCTURE

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SUMMARY

The design of a prestressed concrete roof is described. A special linear elastic analysis of non-prismatic folded structures has been applied. -- The obtained results have been compared to the results deducted from a - small scale model test. Conclusions about the efficency of this type of structures are shown.

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Introduction.

Among the different sport halls built at the Barcelona Football Club, the Hockey Ice Pabellon presents several interesting features.

The rectangular planform of this building is 44.00 x 66.00 m. approximately, and several structural solutions for its roof have been considered. - One of the studied solutions, a non-prismatic folded plate, is here commented with some detail.

Design description

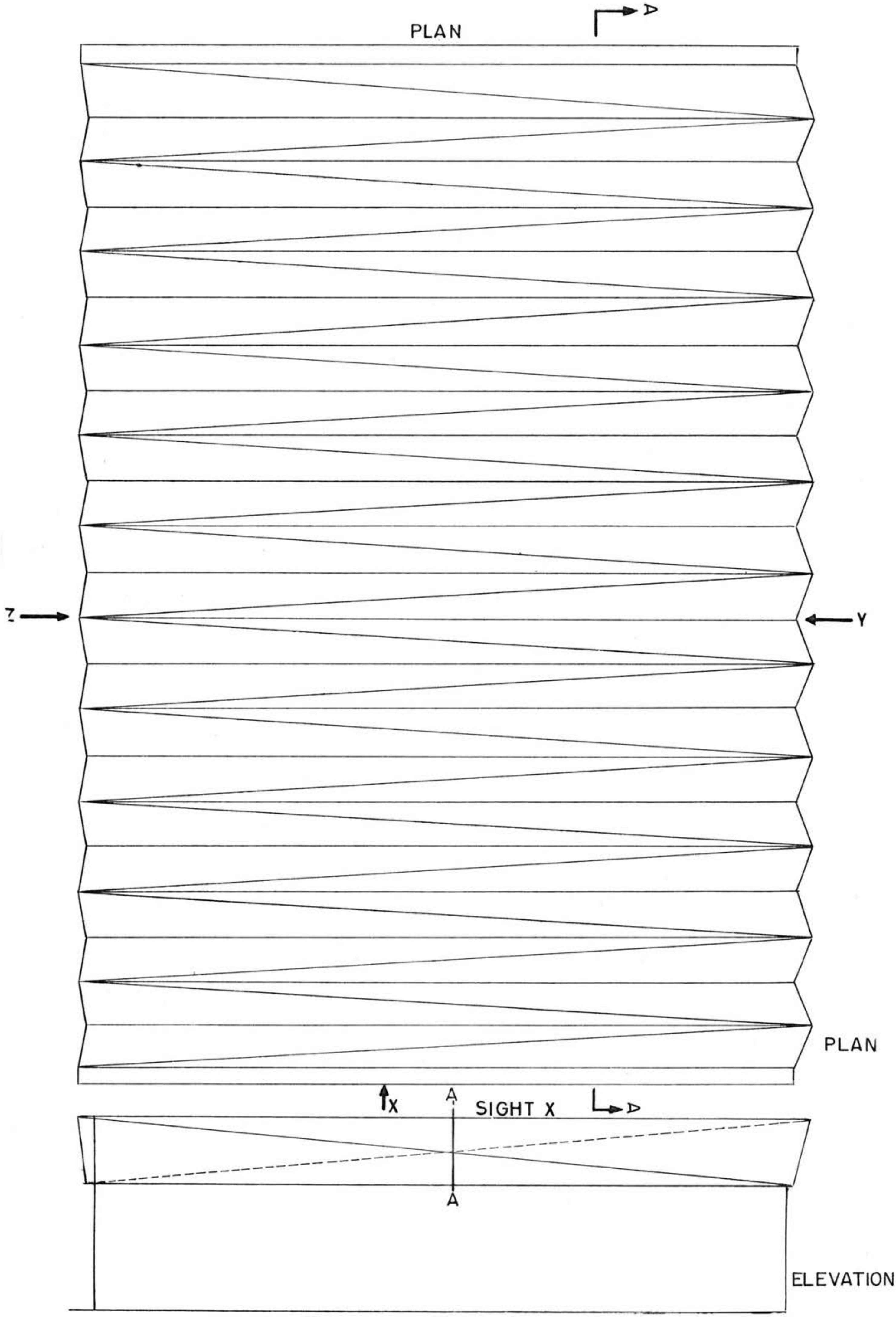
The prestressed concrete roof is formed by nine identical inner non-prismatic folded plate structures, with the following dimensions: 44.25 m. span and 6 m. width. Each of these shells are simply supported on transversal - end gables, and along its longitudinal edges the continuity exists. The two outer non-prismatic folded plate structures, are identical to the inner one except the lateral edges, that are cantilevers of 1,25 m. span (figure 1).

The shell thickness is 0,12 m. and each non-prismatic folded plate is formed by four triangles. Due to analysis and construction considerations, -- the thickness along the intersection of two triangles, have been increased according to figure 2 (section A-A).

The prestressed cable have been designed as shown in figure 3 in order to carry the dead and live loads. The main consideration in its design was - the longitudinal bending stresses. The small transversal flexural stresses have been resisted by means of ordinary steel reinforcement (figure 3).

Analysis

During the selection of the method of the roof analysis the possibility - to use a small easily available digital computer has been an important -- consideration.


PLAN
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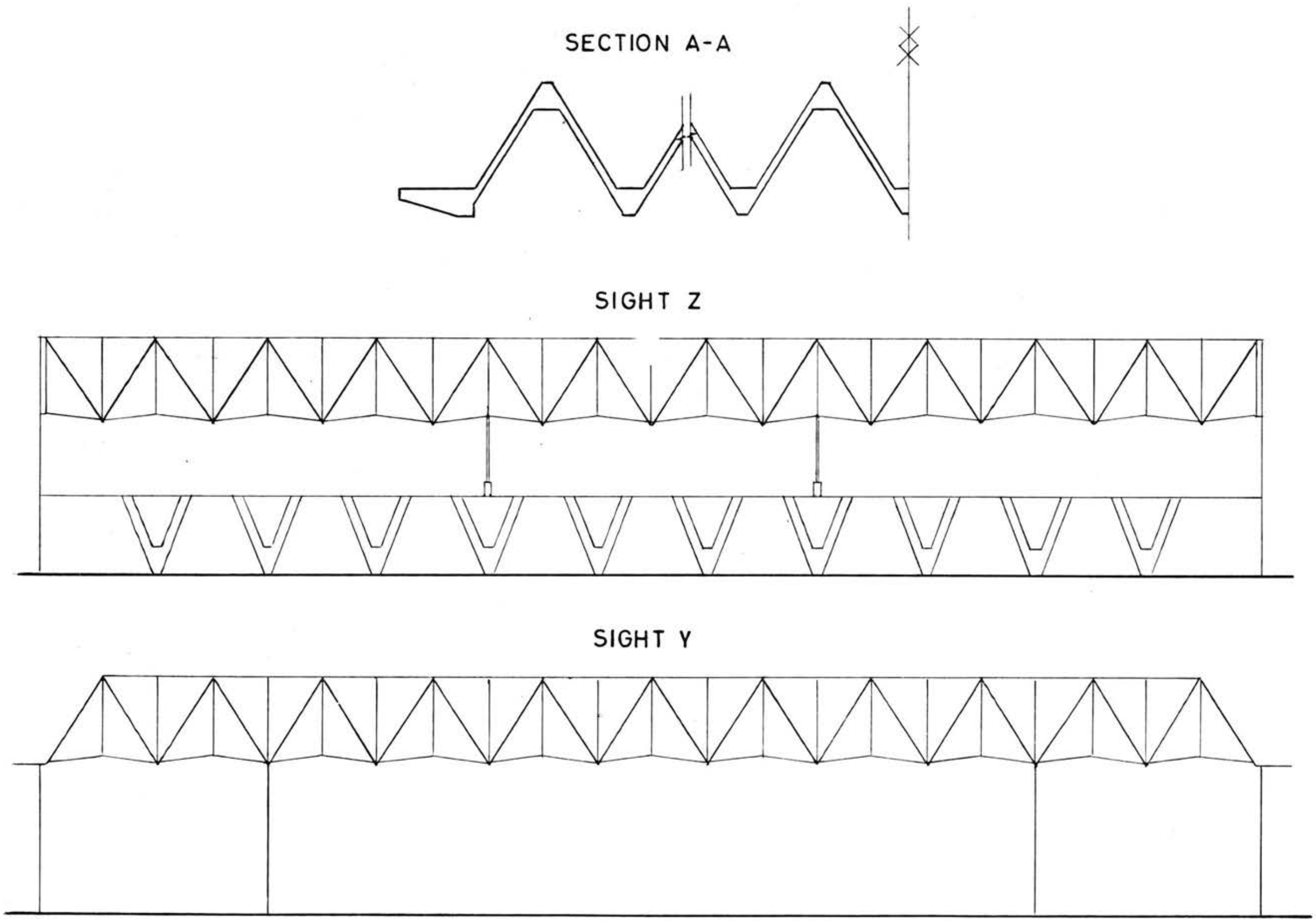
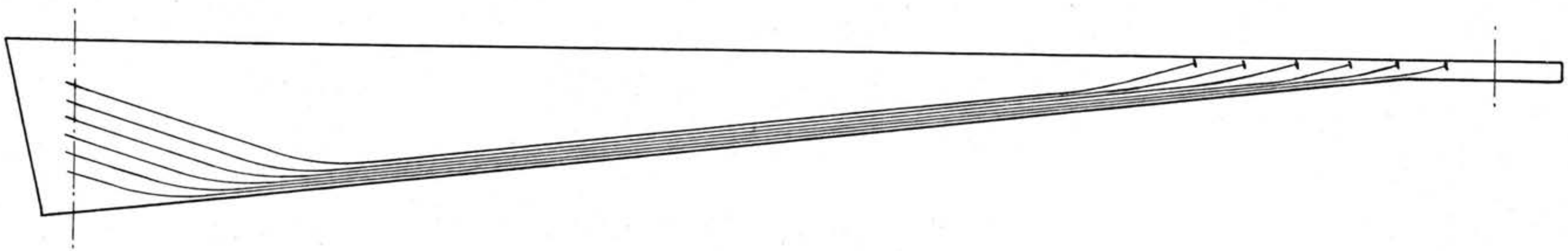


FIGURE 2.-DEFINITION OF THE ROOF

LONGITUDINAL SECTION
PRESTRESSED CABLES



TRANSVERSAL SECTION

PRESTRESSED CABLES   ORDINARY STEEL REINFORCEMENT

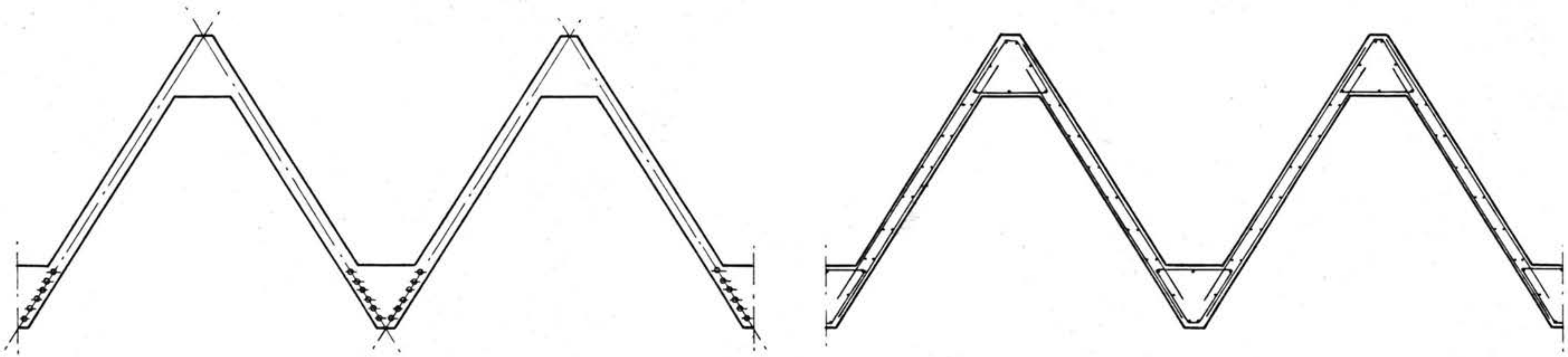


FIGURE 3._ ROOF REINFORCEMENT

For that reason, very general numerical methods as Finite Elements, were avoided. Among the specific methods of analysis of non-prismatic folded plate structures, the method given in the reference (1), wich is an extension of the clasical Yitzhaki analysis is developed. The main steps of the computations are not repeated here but only the introducted changes to the original presentation, namely:

1) Use the Flexibility or Forces method during the transversal analysis, that appears to be more efficient than the previous used Stiffness method.

2) Apply the same computer soubroutine (conjugate beam properties) to the computation of the longitudinal bending moments (M_L) and the deflexion of each plate (W).

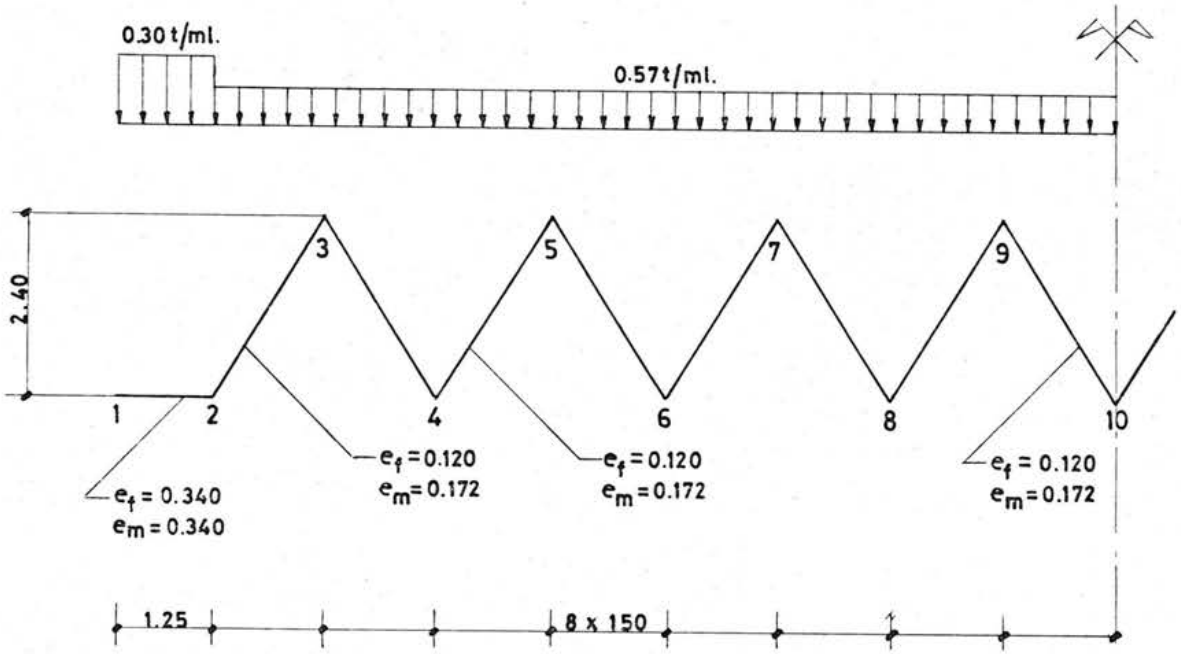
3) In the secondary analysis, that takes into account the effects of ficticions reactions applied to the shell, the following change was considered: Use as reaction representation a parabolic distributed loads between each two adjacent stations insteads of concentrated punctual loads.

4) Into the computer program general boundary conditions have been considered.

The computer program has been written in FORTRAN IV for an IBM-1130/32K computer, using overlay technique.

In order to test the computer calculations the prismatic folded plate of the figure 4, has been analysed using the ordinary folded plate theory (2) L.P.P. (3 terms of the Fourier series) and the general non-prismatic folded plate theory L.P.N.P. (number of stations = 8).

The results obtained from the two methods have agreed satisfactorily for two types of transversal boundary conditions. In the table I these results are shown.



SPAN = 44.34
$E = 1.31 \times 10^6$
$\nu = 0$
UNIFORM LOAD
UNITS: METERS TONS.

FIGURE 4._CROSS SECTION OF THE PRISMATIC FOLDED PLATE STRUCTURE OF THE EXAMPLE

TABLE I

COMPARISON EXAMPLE - RESULTS

A) HYPOTESIS: FREE EDGE 2.

VERTICAL DISPLACEMENTS (mm, ↓+) CENTER TRANSVERSAL SECTION (MIDSPAN)

Edge	1	2	3	4	5	6	7	8	9	10
LPP	188	156	127	130	128	135	137	140	141	141
LPNP	176	150	119	121	119	125	127	130	132	132

HORIZONTAL DISPLACEMENTS (mm, →+) CENTER TRANSVERSAL SECTION (MIDSPAN)

Edge	1	2	3	4	5	6	7	8	9	10
LPP	62.2	62.2	15.5	10.6	8.0	-2.1	2.0	-1.9	2.2	0
LPNP	62.5	62.5	13.6	10.2	7.0	-2.6	1.6	-2.2	0.1	0

LONGITUDINAL BENDING STRESSES (Kg./cm2,) CENTER TRANSVERSAL SECTION (MIDSPAN)

Edge	1	2	3	4	5	6	7	8	9	10
LPP	16.1	58.3	-115.8	90.9	-96.6	105.3	-102.0	108.4	-106.2	108.7
LPNP	12.6	59.2	-113.0	87.2	-93.1	101.6	- 98.3	104.9	-102.7	105.1

TRANSVERSAL BENDING MOMENTS (mt/ml, +) CENTER TRANSVERSAL SECTION (MIDSPAN)

Edge	1	2	3	4	5	6	7	8	9	10
LPP	0.	-0.19	-2.60	0.98	-1.12	0.35	-0.31	-0.07	-0.11	-0.18
LPNP	0.	-0.23	-2.53	1.00	-1.03	0.38	-0.22	-0.03	-0.02	-0.13

SHEAR STRESSES AT EDGES (t/ml, +) SUPPORTS

Edge	1	2	3	4	5	6	7	8	9	10
LPP	0	11.31	1.61	-3.12	-3.98	-2.49	-1.92	-0.80	-0.42	0
LPNP	0	10.85	1.51	-2.96	-3.95	-2.50	-1.93	-0.80	-0.42	0

TABLE I (Contd)

B) HYPOTESIS: VERTICALY SUPPORTED EDGE 2.

VERTICAL DISPLACEMENTS (mm, ↓ +) CENTER TRANSVERSAL SECTION. (MIDSPAN)

Edge	1	2	3	4	5	6	7	8	9	10
LPP	-45	0	52	96	123	137	143	145	145	145
LPNP	-40	0	48	88	113	127	133	135	136	135

HORIZONTAL DISPLACEMENTS (mm. → +) CENTER TRANSVERSAL SECTION (MIDSPAN)

Edge	1	2	3	4	5	6	7	8	9	10
LPP	-41.4	-41.4	42.3	-27.5	15.8	-7.4	2.3	-0.4	-0.2	0
LPNP	-37.5	-37.5	38.9	-25.8	14.9	-7.0	2.3	-0.5	-0.2	0

LONGITUDINAL BENDING STRESSES (Kg./cm2, ←→) CENTER TRANSVERSAL SECTION (MIDSPAN)

Edge	1	2	3	4	5	6	7	8	9	10
LPP	29.2	-3.5	-42.7	75.9	-95.0	-106.3	-109.5	110.8	110.4	110.3
LPNP	27.3	-3.5	-40.4	72.9	-91.5	-102.5	-105.8	107.2	-106.8	106.7

TRANSVERSAL BENDING MOMENTS (mt/ml, +) CENTER TRANSVERSAL SECTION (MIDSPAN)

Edge	1	2	3	4	5	6	7	8	9	10
LPP	0.	-0.19	0.19	0.71	0.34	0.21	0.00	-0.11	-0.16	-0.19
LPNP	0.	-0.23	0.20	0.69	0.38	0.25	0.06	-0.05	-0.09	-0.12

SHEAR STRESSES AT EDGES (t/ml, +) SUPPORTS

Edge	1	2	3	4	5	6	7	8	9	10
LPP	0.	4.47	-4.95	1.87	-1.73	0.29	-0.27	-0.05	0.01	0.
LPNP	0.	3.60	-4.00	1.62	-1.58	0.31	-0.25	-0.03	-0.02	0.

From the table I, the number of stations ($ne = 8$) has been considered suf ficiente from the practical accuracy point of view. Nevertheless in the analysis of the roof this number was increased to $ne = 10$.

The data used in the analysis of the actual non-prismatic folded plate -- roof are given in the table II.

In the table III the computer results of this analysis are presented.

Scale model test

In order to obtain additional information about the shell structural beha viour, experimental analysis was considered. The model test was carried - out at the Laboratorio Central de Ensayos de Materiales de Madrid.

The model scale was 1/24, large enough in order to reproduce without spe- cial difficulties the actual thickness of the shell. In fact the model -- shell thickness was reduced to 5 mm.

The construction of the model has been carried out according to the follo wing steps:

1) An accurate wood model was built. From this previous model the corres-- ponding plaster formworks (molds) were manufactured.

2) The selected model material was a mix of polyester resin and cement. This mixture has been experimented at the Laboratory Central for a long - time and has presented the following elastic constants:

Young modulus $E_M = 73500$ Kp.cm^{-2}

Poisson Ratio $\nu_M = 0.32$

3) The mix was poured into the above mentioned formworks. After a partial material polymerization, the molds were taken out and the model was ther- mosetted in an owen at 50º C during a week.

In the photographs 1 and 2 the finished model is shown.

4) Following the usual experimental practice several material samples we- re prepared simultaneously to the model construction. These specimens we-

TABLE II

DATA OF THE NON-PRISMATIC FOLDED PLATE ROOF

(Notation is given in the appendix A.)

1.1: General data.

$I = 9 \quad N = 10 \quad A = 10 \quad E = 1310000$ t/m2.

1.2. Plate longitudinal data.

$L(i) = 44.34$ m. $\quad i = 1, 2, \dots 9.$

$\beta(i) = 4.44$ º $\quad i = 2, 5, 6, 9.$

$\beta(i) = -4.44$ º $\quad i = 3, 4, 7, 8.$

1.3 Plate 1.

$h(\alpha, 1) = 1.25 \quad v(\alpha, 1) = 0. \quad e_m(\alpha, 1) = 0.34, \; e_f(\alpha, 1) = 0.34$

$\alpha = 1, 2, \dots 10$

Plates 2, 5, 6 and 9.

$h(1, i) = 0.150$	$v(1, i) = \pm 0.240$	$e_m(1, i) = 0.299$
$h(2, i) = 0.445$	$v(2, i) = \pm 0.710$	$e_m(2, i) = 0.265$
$h(3, i) = 0.745$	$v(3, i) = \pm 1.190$	$e_m(3, i) = 0.233$
$h(4, i) = 1.050$	$v(4, i) = \pm 1.680$	$e_m(4, i) = 0.203$
$h(5, i) = 1.345$	$v(5, i) = \pm 2.160$	$e_m(4, i) = 0.182$
$h(6, i) = 1.645$	$v(6, i) = \pm 2.630$	$e_m(4, i) = 0.165$
$h(7, i) = 1.960$	$v(7, i) = \pm 3.120$	$e_m(4, i) = 0.152$
$h(8, i) = 2.250$	$v(8, i) = \pm 3.610$	$e_m(4, i) = 0.145$
$h(9, i) = 2.550$	$v(9, i) = \pm 4.100$	$e_m(4, i) = 0.140$
$h(10, i) = 2.850$	$v(10, i) = \pm 4.600$	$e_m(4, i) = 0.137$

and $e_f(\alpha, i) = 0.120$

where $\alpha = 1, 2, \dots 10 \quad i = 2, 5, 6$ and 9.

The sign + corresponds to i = 2, and 6 and the sign − to i = 5 and 9 in the values of $v(\alpha, i)$

TABLE II (Cont.)

Plates 3, 4, 7 and 8.

Let be $i = 4$ and 8 or and 7, and
$i_1 = 2$ and 6 or 5 and9, then

$$h(i,\alpha) = h(i_1, 11-\alpha)$$
$$v(i,\alpha) = v(i_1, 11-\alpha)$$
$$e_m(i,\alpha) = e_m(i_1, 11-\alpha)$$
$$e_f(i,\alpha) = e_f(i_1, 11-\alpha)$$
$$\alpha = 1, 2, \dots 10$$

1.4. Loads

Uniform distributed loads in plates.

$P(1) = -300 \text{ kp m}^{-2}$

$P(i) = -570 \text{ kp m}^{-2} \quad i = 2, 3, \dots 9.$

TABLE III.

RESULTS OF THE NON-PRISMATIC FOLDED PLATE ROOF (Notation is given in the appendix A).

SHEAR STRESSES (TON.).

STATION	EDGE 2.	EDGE 3.	EDGE 4.	EDGE 5.	EDGE 6.	EDGE 7.	EDGE 8.	EDGE 9.
2	-17.50	-14.76	33.69	42.40	-10.58	-55.18	6.56	50.41
3	-4.35	-20.42	63.77	31.52	-0.79	-35.08	8.93	32.17
4	21.25	-21.85	73.94	15.90	9.77	-13.48	8.75	13.24
5	47.37	-21.88	66.04	6.51	19.11	-1.83	7.21	3.81
6	66.38	-17.93	44.58	-2.34	26.40	5.77	5.03	-3.01
7	73.61	-11.44	15.21	-16.01	30.28	14.89	2.58	-13.11
8	63.79	-0.84	-17.86	-40.20	29.10	32.27	-0.12	-33.08
9	37.13	3.19	-44.86	-65.33	21.00	47.79	-2.87	-52.16

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TABLE III. (Cont.)

LONGITUDINAL BENDING STRESSES (KG/CM^2).

STATION		PLATE 1.	PLATE 2.	PLATE 3.	PLATE 4.	PLATE 5.	PLATE 6.	PLATE 7.	PLATE 8.	PLATE 9.
2	S1	-8.05	-0.15	2.53	13.14	-10.44	-37.13	-3.06	21.47	-8.40
	S2	-0.18	2.61	11.76	-10.57	-37.14	-2.92	21.27	-8.53	-36.86
3	S1	1.23	-3.53	-6.23	34.23	-44.84	24.86	-45.50	59.77	-52.48
	S2	-3.28	-6.26	33.45	-44.65	25.14	-45.76	59.73	-52.25	32.87
4	S1	13.10	-3.52	-17.78	51.82	-72.63	69.25	-80.64	88.52	-87.04
	S2	-3.10	-17.85	51.94	-72.51	69.59	-80.79	88.57	-86.92	80.48
5	S1	23.96	-2.12	-27.62	61.12	-84.35	89.43	-98.42	101.85	-103.21
	S2	-1.66	-27.69	61.88	-84.31	89.78	-98.45	101.94	-103.18	101.57
6	S1	28.92	1.94	-34.74	60.64	-80.81	91.72	-99.79	99.37	-102.81
	S2	2.31	-34.79	61.66	-80.85	92.01	-99.75	99.47	-102.84	103.99
7	S1	29.95	4.59	-34.93	47.14	-62.46	78.84	-84.45	78.26	-85.91
	S2	4.68	-34.92	48.15	-62.62	78.97	-84.32	78.35	-86.05	90.59
8	S1	24.31	6.01	-26.95	15.81	-29.12	51.68	-50.86	31.04	-50.88
	S2	5.70	-26.90	16.57	-29.42	51.52	-50.65	31.11	-51.12	61.58
9	S1	13.55	4.58	-14.60	-28.31	9.76	16.33	-8.29	-37.19	-7.20
	S2	3.92	-14.59	-28.55	9.92	15.70	-8.43	-37.19	-7.06	22.59

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TABLE III (Cont.)

TRANSVERSAL BENDING MOMENTS (MT/ML).

STATION	EDGE 1.	EDGE 2.	EDGE 3.	EDGE 4.	EDGE 5.	EDGE 6.	EDGE 7.	EDGE 8.	EDGE 9.	EDGE 10.
2	0.00	-0.23	1.80	-0.64	-0.86	1.80	-0.04	-0.29	-0.59	1.95
3	0.00	-0.23	1.81	-0.46	-1.19	2.96	-0.41	-0.16	-0.97	3.00
4	0.00	-0.23	1.37	-0.31	-0.72	1.89	-0.20	-0.30	-0.59	1.75
5	0.00	-0.23	0.63	0.02	-0.04	0.65	0.00	-0.18	-0.18	0.45
6	0.00	-0.23	-0.29	0.68	0.40	-0.07	-0.14	0.51	-0.15	-0.22
7	0.00	-0.23	-1.22	1.63	0.53	-0.31	-0.63	1.80	-0.50	-0.31
8	0.00	-0.23	-1.79	2.38	0.51	-0.26	-1.04	3.01	-0.82	-0.13
9	0.00	-0.23	-1.55	1.40	0.71	-0.41	-0.68	1.92	-0.43	-0.25

.../.

TABLE III. (Cont.)

D E F L E C T I O N S $(10^{-3}$ M).

STATION	PLATE 1.	PLATE 2.	PLATE 3.	PLATE 4.	PLATE 5.	PLATE 6.	PLATE 7.	PLATE 8.	PLATE 9.
2	-7.14	-3.79	13.55	-20.71	26.44	-27.47	29.77	-30.64	32.43
3	-15.00	-7.96	26.59	-40.26	55.70	-58.92	58.20	-59.75	67.67
4	-22.18	-11.75	37.83	-56.33	77.26	-82.73	81.99	-83.92	93.47
5	-27.02	-14.28	45.55	-66.19	86.33	-93.38	97.33	-99.34	104.50
6	-28.25	-14.98	47.95	-67.51	83.01	-90.67	100.85	-102.67	100.94
7	-25.61	-13.57	43.49	-58.80	69.52	-76.69	90.18	-91.60	85.17
8	-19.34	-10.23	31.74	-40.56	48.96	-54.58	65.16	-66.05	60.59
9	-10.37	-5.48	15.20	-17.74	24.83	-27.96	31.12	-31.47	31.06

re-tested under simple bending stresses in order to obtain the values of E_M and ν_M.

5) The model test stresses were measured during the testing at 126 points, according to the figures 5 and 6. These points corresponding to 52 triangular rosettes and 74 longitudinal strain-gages.

6) The considered loading condition was the self-weight and was simulated by means extensional rubber tubes separated in plan 6.25 cms. By applications of three hydraulic jacks over a horizontal strong steel frame the above mentioned rubber tubes were under tension producing the desired simulated dead load in the model.

Photographs 3 and 4 shows the loading technique used.

7) The shell actual boundary conditions were represented into the model according to photograph 4 and its importance was evident during the testing.

8) The distributed dead load considered in the prototype was

$$p_p = 0.055 \text{ kp cm}^{-2}$$

The model acting load in each rubber spring is 1 kps, and the equivalent distributed load P_M is

$$P_M = \frac{1}{6.25^2} = 0.256 \text{ kp cm}^{-2}$$

And the stress ratio is

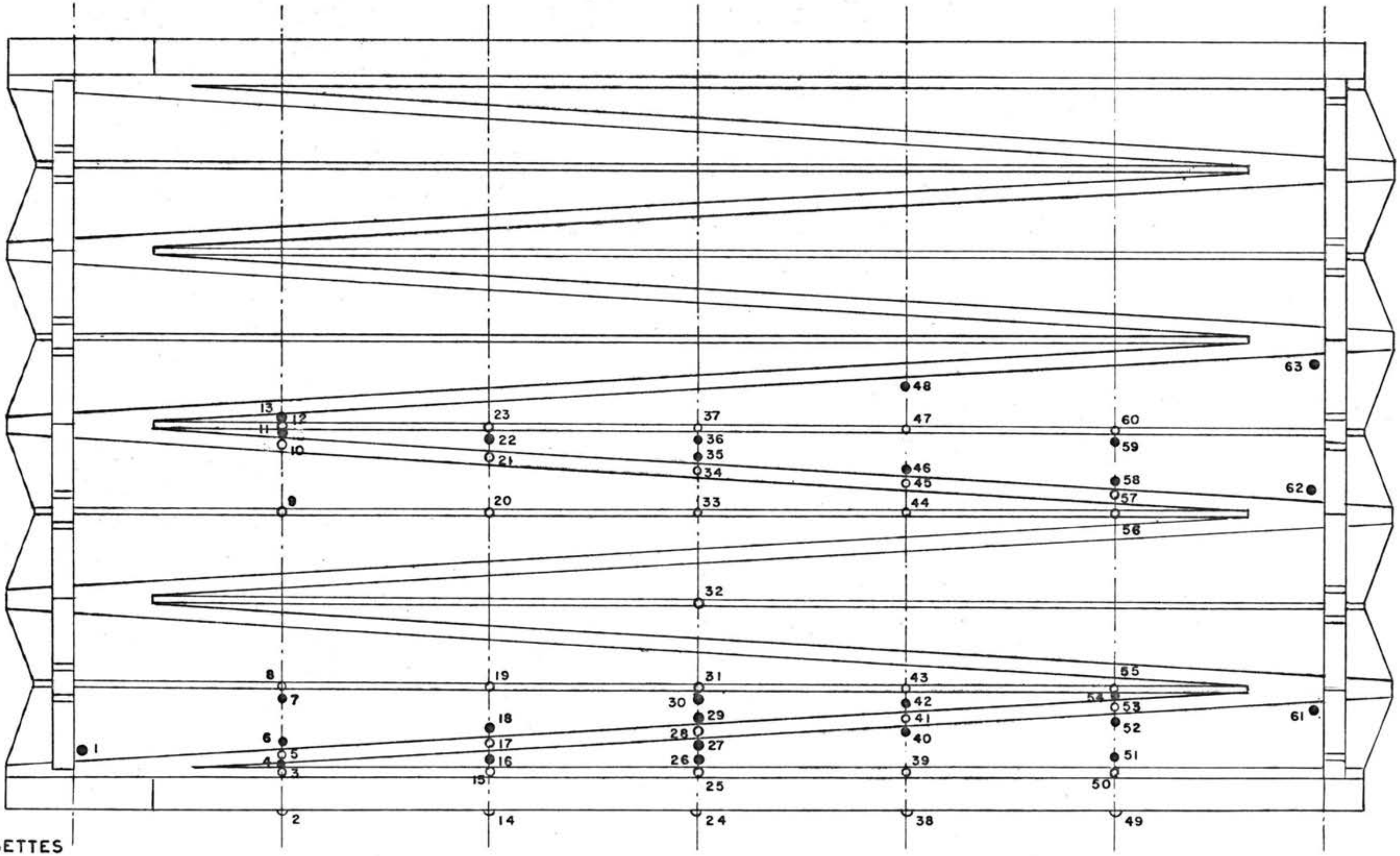
$$\frac{\sigma_P}{\sigma_M} = \frac{P_P}{P_M} = 2.15$$

The deflection ratio is

$$\frac{f_p}{f_M} = \frac{\epsilon_P L_P}{\epsilon_M L_M} = \frac{\sigma_P L_P E_M}{\sigma_M L_M E_P} = 29.2$$

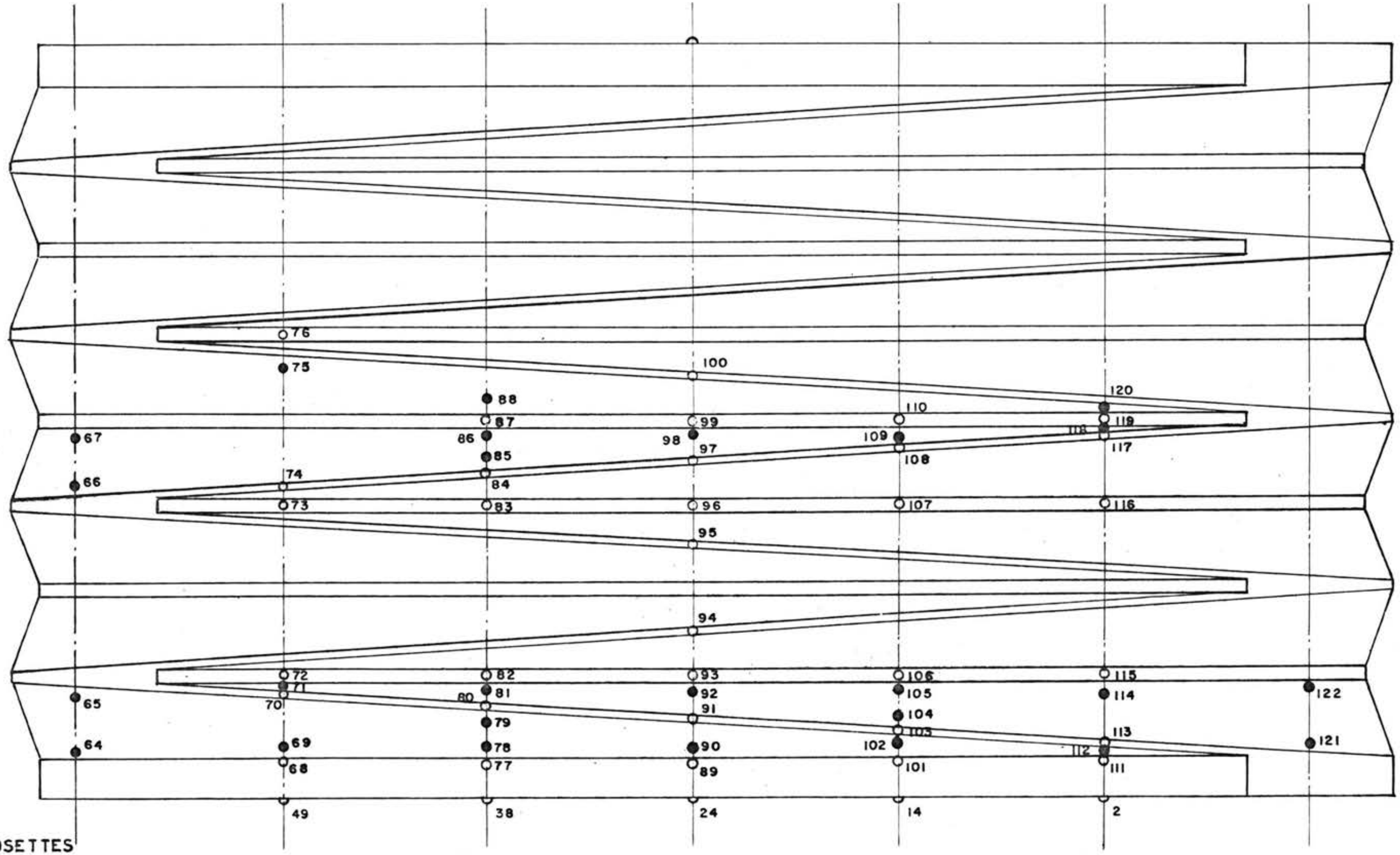
It was assumed $E_p = 130\,000 \text{ kp cm}^{-2}$

The above formula are valid only if $\nu_P = \nu_M$. Some experience and engineering judgement are necesary in order to obtain an accurated model results interpretation.



● ROSETTES
○ STRAIN-GAUGES

FIGURE 5_MODEL INNER FACE_SITUATION OF STRAIN-GAUGES AND ROSETTES



● ROSETTES

○ STRAIN-GAUGES

FIGURE 6.- MODEL OUTER FACE.- SITUATION OF STRAIN-GAUGES AND ROSETTES

The model stress were obtained from the strains by the following formulae

$$\sigma_{1M} = \frac{E_M}{1-\nu_M^2}\left[\epsilon_{1M} + \nu_M \epsilon_{2M}\right]$$

$$\sigma_{2M} = \frac{E_M}{1-\nu_M^2}\left[\epsilon_{2M} + \nu_M \epsilon_{1M}\right]$$

$$\tau_{12M} = \sigma_{3M} = \frac{E_M}{1+\nu_M}\left[\epsilon_{3M} - \frac{\epsilon_{1M}+\epsilon_{2M}}{2}\right]$$

were ϵ_{1M}, ϵ_{2M} and ϵ_{3M} are the strains measured by the rosettes of angle 45º. In the figures 7 and 8 the prototype stresses are shown and in the figure 9 the normal displacements are also given.

Conclusions

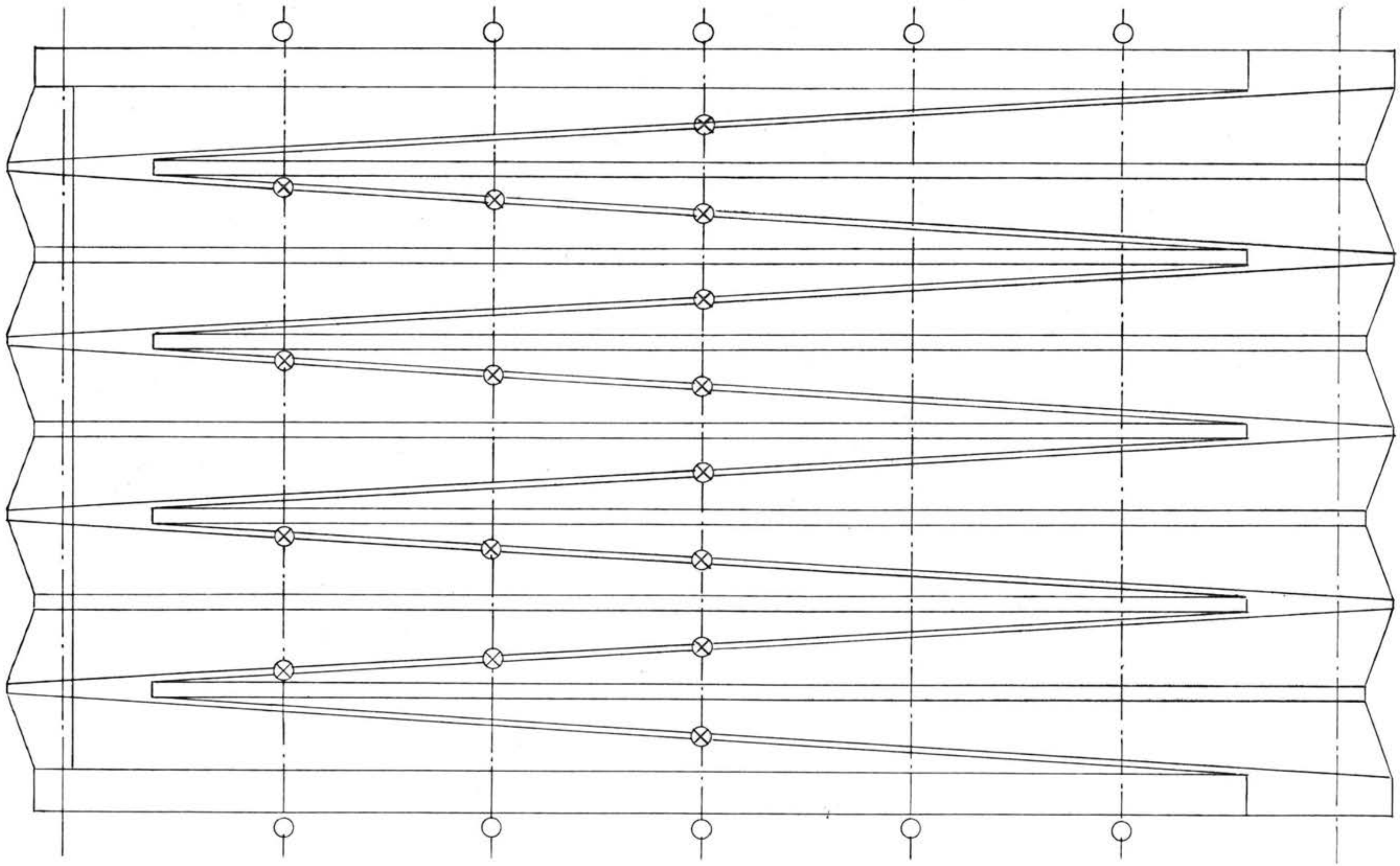
From the experience gained from the design of this type of structures (3) some provisional conclusions can be drawn:

1) The non-prismatic folded plate roof represents a very simple structure from the design point of view.

2) The presented method of numerical analysis, very specialised for this type of structures. seems to be very suitable from the economical and -- computation point of view. The accuracy of the obtained results are enough for practical purposes.

3) The construction and testing of the elastic model of this type of - structures does not offer any new difficulty with respect to another shell structures. rather it seems offer more constructional advantages due to its geometric simplicity.

4) A comparative study of the results from the numerical analysis and model test shows a quite good agreement between them. Some small differences of these results can be explained due to the sensitivity of the model to the accurately representation of the existing boundary conditions.



○ HORIZONTAL TRASDUCTER

⊗ VERTICAL

FIGURE 7.– MODEL OUTER FACE – SITUATION OF DEFLECTION TRASDUCTERS

POINT	STRESSES (kp.cm^{-2}) DIRECTION 1 (σ_1)	DIRECTION 2 (σ_2)	DIRECTION 3 (τ)
1	95	29	-29
2	-12		
3	9		
4	0	0	8
5	-2		
6	74	22	-14
7	-10	14	-5
8	-24		
9	30		
10	-17		
11	30	-4	-4
12	-22		
13	34	-5	-4
14	3		
15	1		
16	-10	-8	-14
17	-10		
18	66	13	3
19	51		
20	86		
21	-48		
22	57	18	9
23	76		
24	28		
25	-7		
26	-35		
27	13	-17	-2
28	-19		
29	25	-2	-3
30	22	41	-6
31	68		
32	86		

POINT	STRESSES (kp.cm^{-2}) DIRECTION 1 (σ_1)	DIRECTION 2 (σ_2)	DIRECTION 3 (τ)
33	115		
34	-65		
35	10	47	12
36	1	39	6
37	108		
38	40		
39	-3		
40	-56	-44	3
41	-21		
42	48	19	-1
43	40		
44	72		
45	72		
46	-25	-55	2
47	83		
48	-26	-58	1
49	8		
50	13		
51	9		
52	-60	-37	11
53	-18		
54	24	-12	18
55	-19		
56	-23		
57	21		
58	-26	-23	13
59	4	18	-4
60	30		
61	0	-8	-2
62	1	2	5
63	1	-3	6

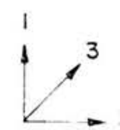


FIGURE 8_ PROTOTYPE STRESSES_ INNER FACE (kp.cm^{-2})

POINT	STRESSES (kp.cm^{-2}) DIRECTION 1 (σ_1)	DIRECTION 2 (σ_2)	DIRECTION 3 (τ)
64	-31	-33	16
65	7	-13	5
66	4	7	7
67	-12	2	8
68	5		
69	-18	-7	2
70	-18		
71	4	-20	26
72	-11		
73	-18		
74	-12		
75	-3	15	7
76	21		
77	-6		
78	6	-12	8
79	53	-4	7
80	-30		
81	11	-4	6
82	12		
83	20		
84	-73		
85	25	37	5
86	4	33	0
87	46		
88	5	36	-5
89	-9		
90	54	5	13
91	-26		
92	-20	22	-7
93	33		
	-78		
95	-97		
96	50		
97	-97		
98	4	30	-2
99	52		
100	-100		
101	-5		
102	-30	-19	0
103	-17		
104	-55	-27	-4
105	2	32	-10
106	31		
107	46		
108	-74		
109	-11	-26	-3
110	16		
111	-1		
112	-5	1	1
113	-10		
114	14	23	-4
115	17		
116	21		
117	-15		
118	-9	-14	-16
119	16		
120	-11	-18	-17
121	-40	9	-12
122	-7	10	-11

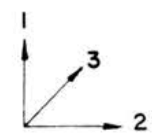
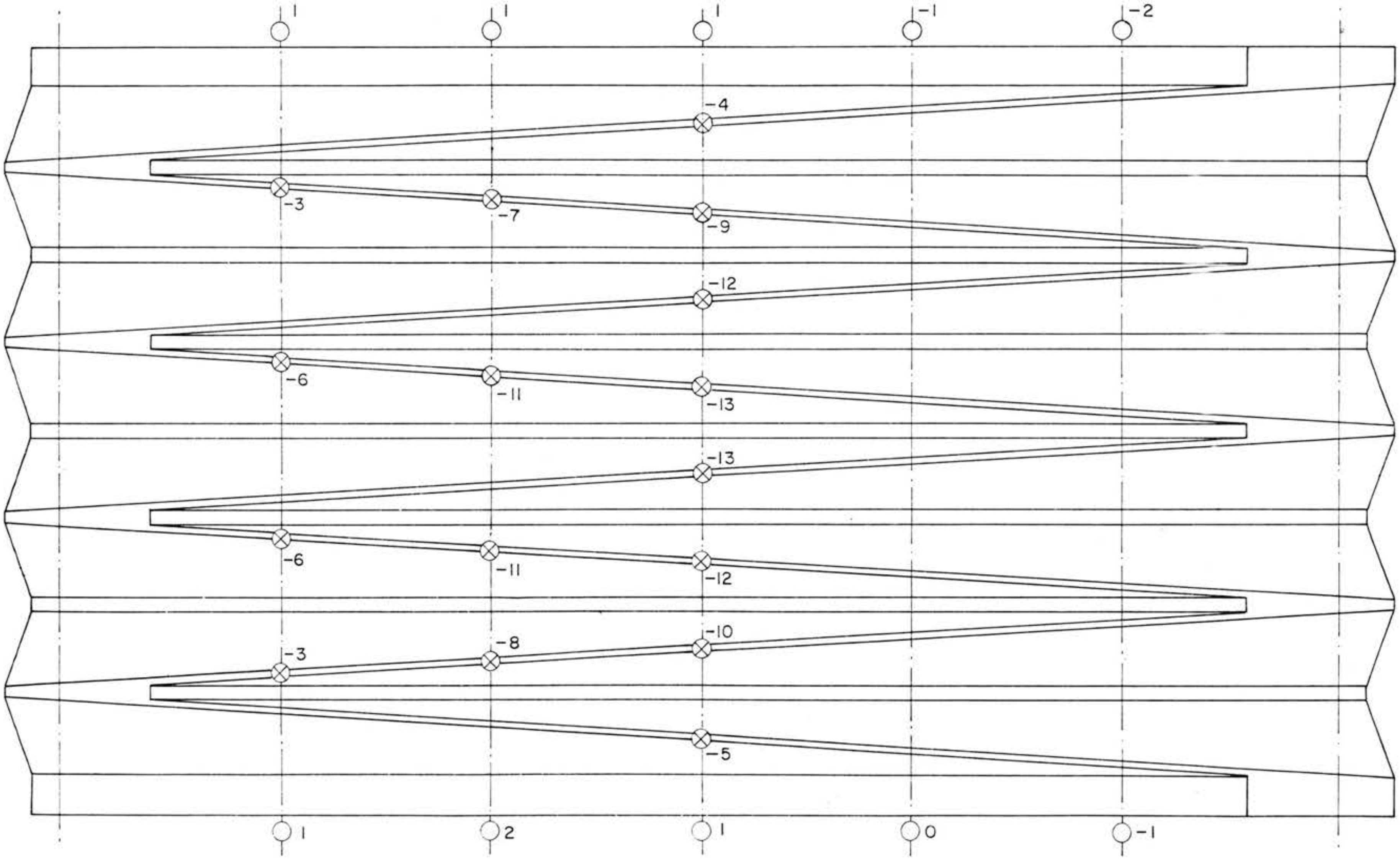


FIGURE 9._PROTOTYPE STRESSES._ OUTER FACE (kp.cm^{-2})



○ HORIZONTAL TRASDUCTER

⊗ VERTICAL

FIGURE 10._ PROTOTYPE DISPLACEMENTS (mm.)

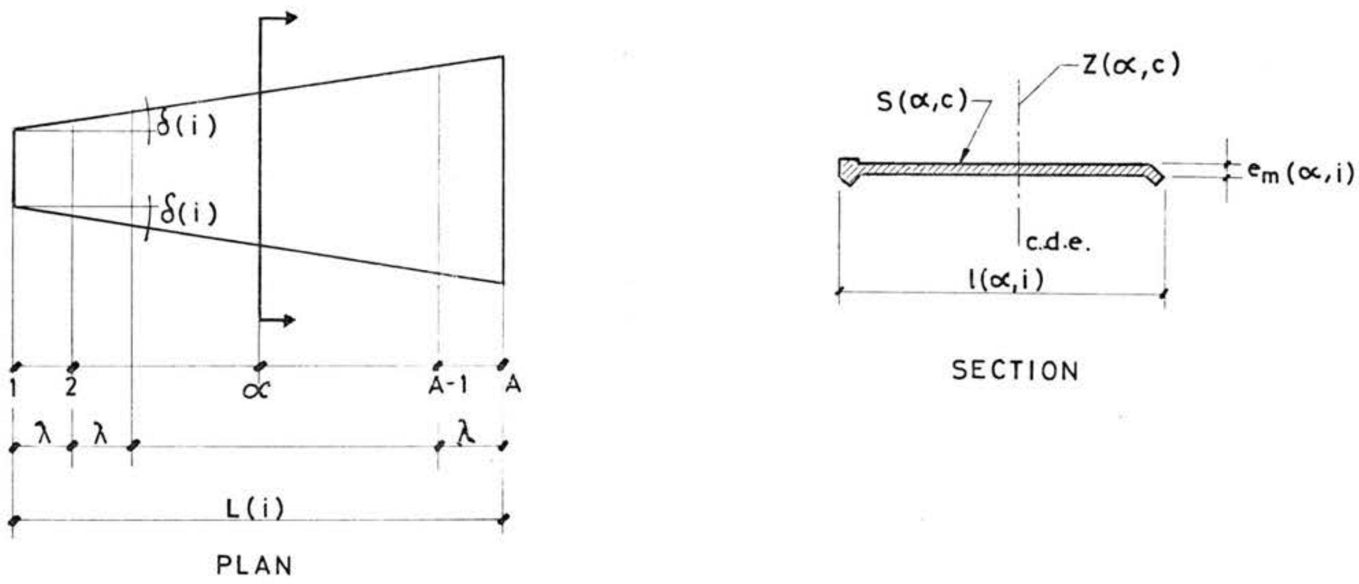


FIGURE 11.– DEFINITION OF THE PLATE no. i

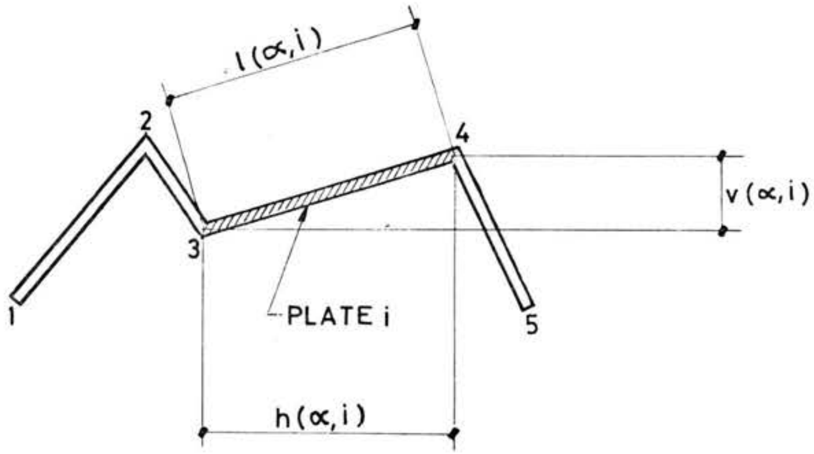


FIGURE 12.- TRANSVERSAL SECTION OF THE PLATE i

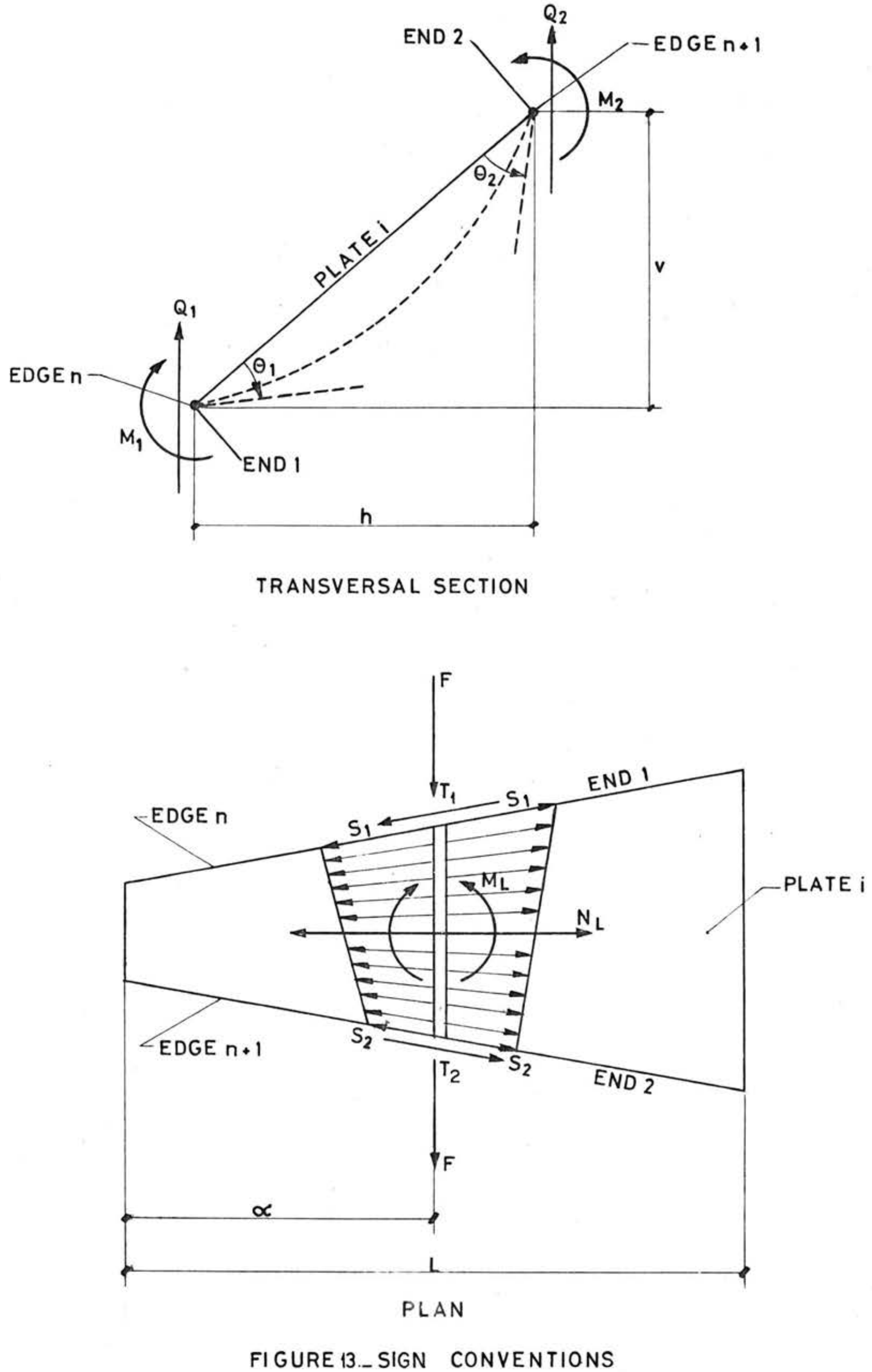
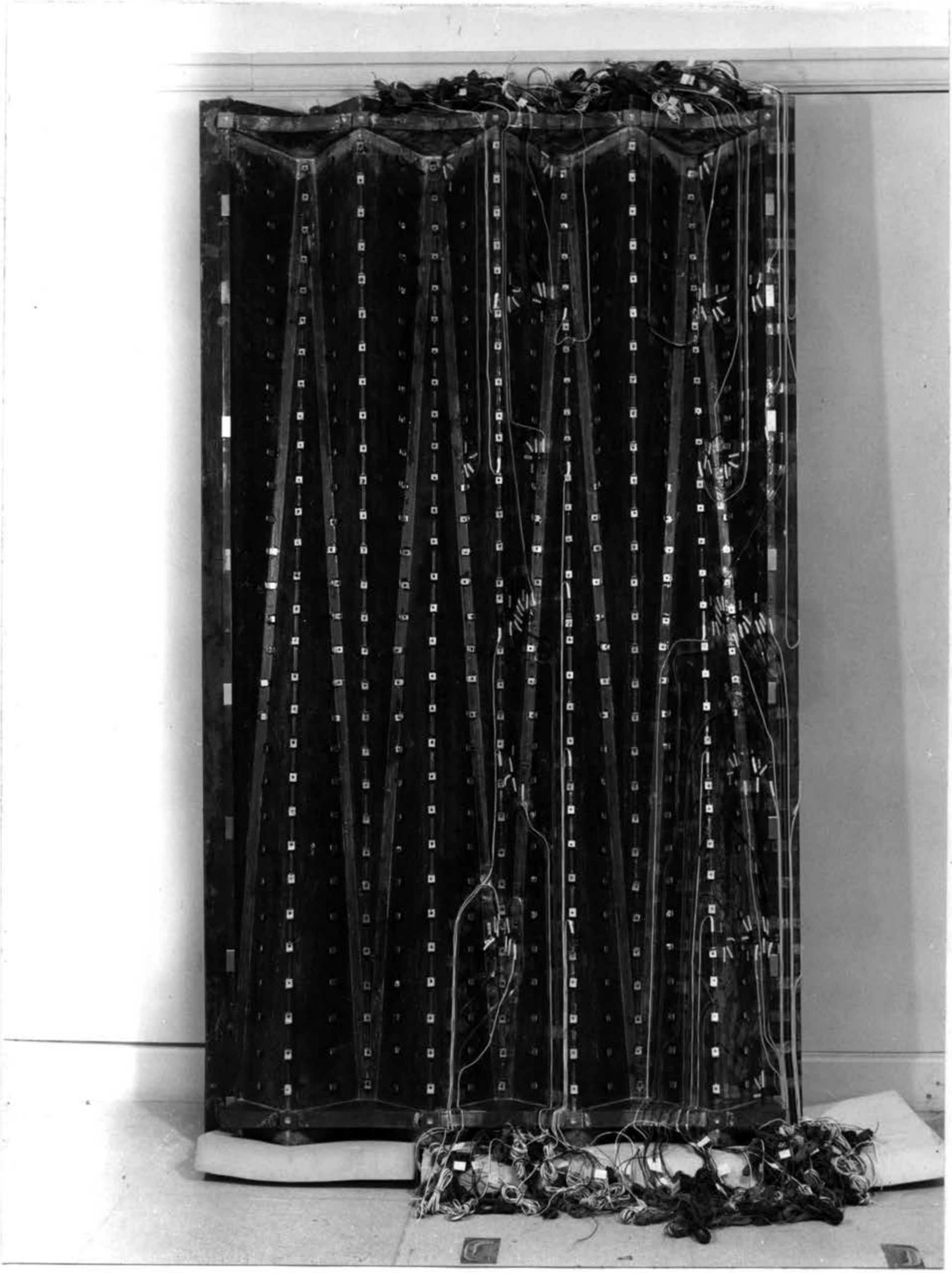
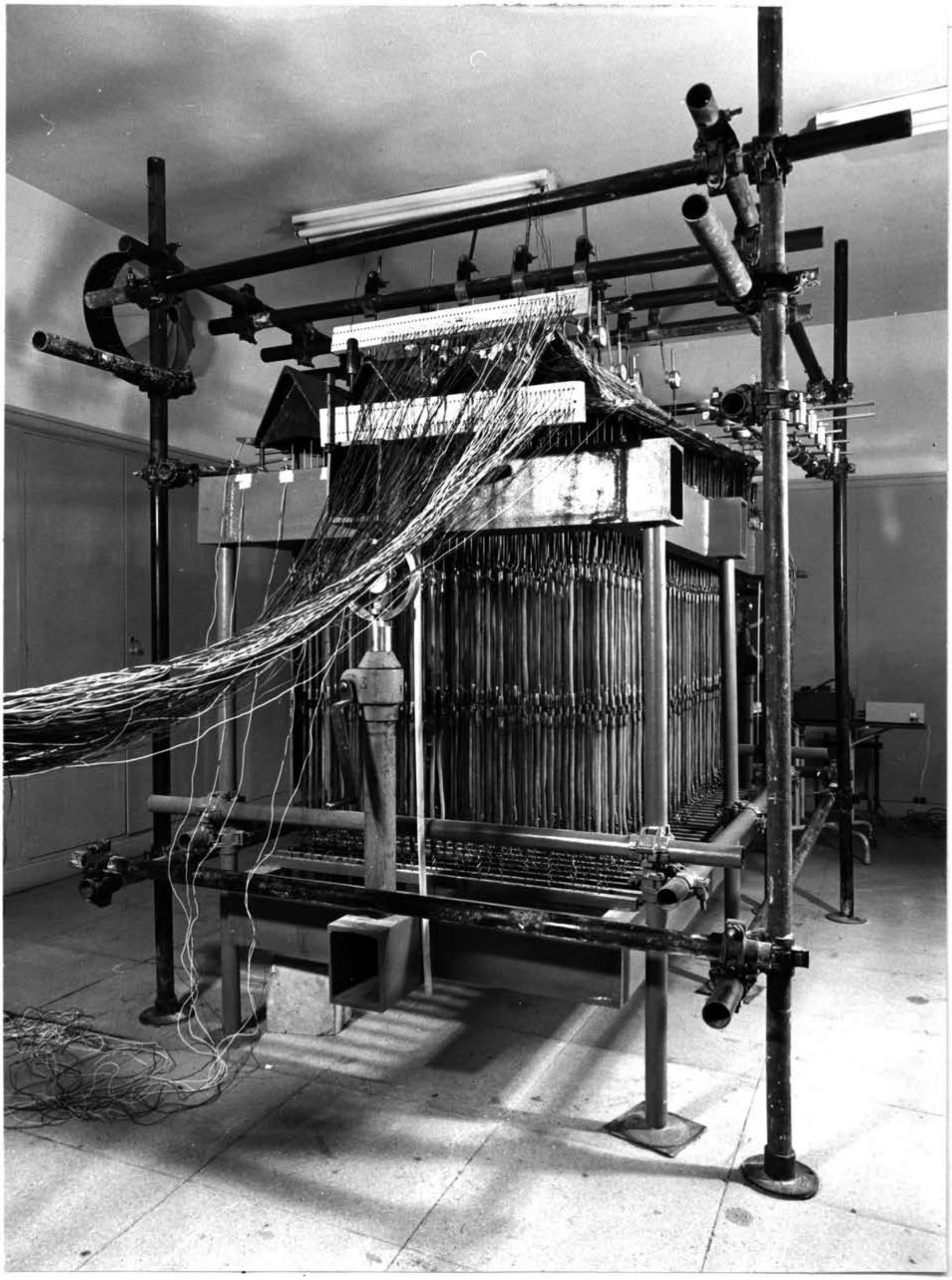


FIGURE 13.– SIGN CONVENTIONS

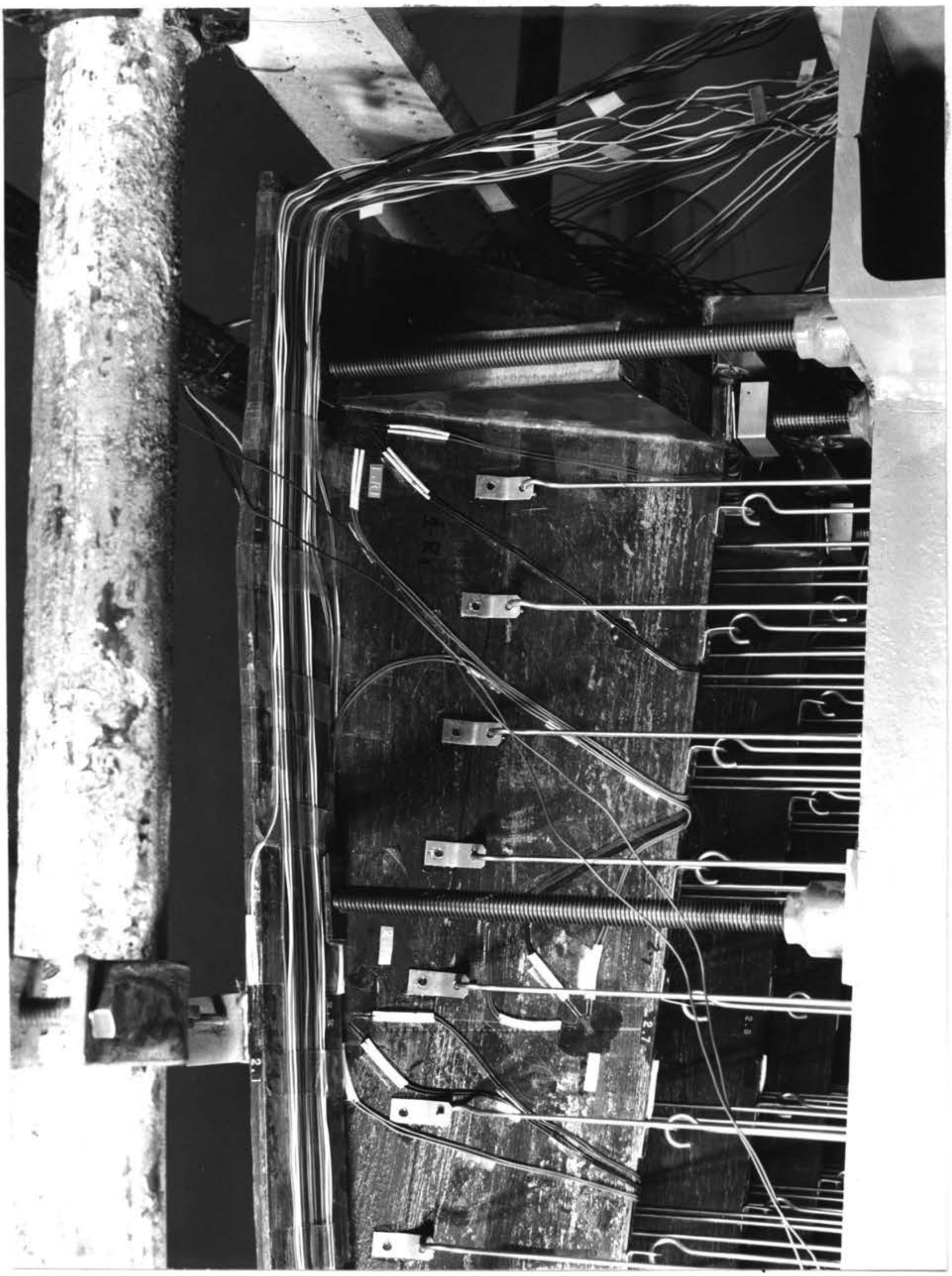


PHOTOGRAPH 1.- Model inner face. Situation of strain gages and rosettes.

PHOTOGRAPH 2.- Model outer face. Situation of Strain gages and rosettes.



PHOTOGRAPH 3.- Loading System.



PHOTOGRAPH 4.- Loading System and Boundary conditions modelling.Details.

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Appendix A.-

Notations Computer Program Description.

1. Input Data
   1.1. General Data
      I = number of plates
      N = number of edges (N = I + 1)
      A = number of stations
      E = Modulus of elasticity

   1.2. Plate data

      1.2.1. Longitudinal (fig 10)
         i = identification number of each plate
         L(i) = span of the plate nº. i
         β(i) = angle of the plate nº. i

      1.2.2. Transversal (fig 11)

         α = station or transversal section at position α
         For each plate nº i at the station α is given:

         $h(\alpha,i)$ = horizontal projection
         $v(\alpha,i)$ = vertical projection
         $e_f(\alpha,i)$ = slab thickness (flexural behaviour)
         $e_m(\alpha,i)$ = plate thickness (membane behaviour)
         $S(\alpha,i)$ = area
         $Z(\alpha,i)$ = strengh modulus

Note.- If the section α of the plate nº. i is rectangular, then

$$S(\alpha,i) = \ell(\alpha,i)\, e_m(\alpha,i)$$

$$Z(\alpha,i) = \frac{1}{6} S(\alpha,i)\, \ell(\alpha,i)$$

were

$$\ell(\alpha,i) = \left[h(\alpha,i)^2 + v(\alpha,i)^2 \right]^{\frac{1}{2}}$$

   1.3. Load definition

      1.3.1. Self weight
         $\gamma(i)$ = unit density of the plate nº. i

      1.3.2. Prestressing
         $\nu^o(\alpha,i)$ = axial force acting at the station of the plate nº i
         $\mu_L^o(\alpha,i)$ = bending moment acting at the station of the plate nº i

1.3.3. Uniform distributed loading

$p(i)$ = vertical uniform distributed loading for unit horizon tal projected area of the plate no.i.

1.3.4. Uniform distributed loading along edges

$q(n)$ = vertical uniform distributed loading for unit of length of edge no. n.

1.3.5. Concentrated punctual loads

$R^{o}(\alpha,n)$= punctual load acting at the station of the edge n.

1.4. Boundary conditions.

n = edge number
$n\theta(n)$= boundary condition code for the rotation along the edge N
$nu_{L}(n)$= boundary condition code for the horizontal displaccement
$nu_{n}(n)$ of the edge n.

Note.- If the boundary condition code is equal 0 or 1 means unconstrained or contrained displacement respectively.

2. Computer results (fig 1.2)

$\theta_{k}(\alpha,i)$ = rotation along end k (k = 1,2) of the plate i at the station α
$Q_{k}(\alpha,i)$ = reduced shear per unit length of the end k (K = 1,2) of the pla te i at the station α
$M_{T}(\alpha,n)$= transversal bending moment per unit length of the edge n at the station α
$M_{L}(\alpha,i)$= longitudinal bending moment at the station of the plate i asuming its structural behaviour as a simply supported beam.
$N_{L}(\alpha,i)$= longitudinal axial force at the station of the plate i.
$S_{k}(\alpha,i)$= normal stress acting along end k (k =1,2) of the plate i and due to $M_{L}(\alpha,i)$.
$T_{k}(\alpha,i)$= total shear acting along the end k(k = 1,2) of the plate i at - the station
$W(\alpha,i)$ = beam displacement of the plate i at the section
$\lambda_{k}(\alpha,i)$= normal displacement of the end k (k = 1,2) of the plate i at the section